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उत्प्लव मार्गों के ऊर्जा डिसिपेटरों के संरचनात्मक डिजाईन — मापदण्ड

(पहला पुनरीक्षण)

Indian Standard

STRUCTURAL DESIGN OF ENERGY DISSIPATORS FOR SPILLWAYS — CRITERIA

(First Revision)

ICS 93.16

BUREAU OF INDIAN STANDARDS
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Dams and Spillways Sectional Committee, WRD 09

FOREWORD

This Indian Standard (First Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Dams and Spillways Sectional Committee had been approved by the Water Resources Division Council.

This standard covering the structural design of energy dissipators for spillways was first published in 1985. This revision incorporates the latest practices being followed in the field, the major changes being in respect of design of floor slab anchorages, anchorage for spillway bucket and design of reinforcement for bucket teeth.

The design of downstream protection works or energy dissipators below hydraulic structures occupies a vital place in the design and construction of dams, weirs, barrages and outlets. The problem of designing energy dissipators is essentially of reducing high velocity flow to a velocity low enough to minimize erosion of natural river bed. This reduction in velocity may be accomplished by any, or a combination of the following, depending upon the head, discharge intensity, tail water conditions and the type of bed rock or the bed material:

a) Hydraulic jump type stilling basins:
   1) Horizontal apron type, and
   2) Sloping apron type.
b) Jet diffusion and free jet stilling basins:
   1) Jet diffusion basins,
   2) Free jet stilling basins,
   3) Hump stilling basins, and
   4) Impact stilling basins.
c) Bucket type dissipators:
   1) Solid and slotted roller buckets; and
   2) Trajectory buckets (ski jump, flip, etc).
d) Interacting jets and other special type of stilling basins.

In India, hydraulic jump type stilling basins and bucket type energy dissipators are generally used for dissipation of energy depending on condition of downstream tail water. Indian Standards had already been published for criteria for hydraulic design of these two types of energy dissipators as under:

IS No.	*Title*
4997 : 1968	Criteria for design of hydraulic jump type stilling basins which with horizontal and sloping apron
7365 : 2010	Criteria for hydraulic design of bucket type energy dissipators (*second revision*)

For the purpose of deciding whether a particular requirement of this standard is complied with the final value, observed or calculated expressing the result of test or analysis shall be rounded off in accordance with IS 2 : 1960 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

STRUCTURAL DESIGN OF ENERGY DISSIPATORS FOR SPILLWAYS — CRITERIA

(First Revision)

1 SCOPE

This standard lays down criteria for structural design of various components of hydraulic jump type stilling basins and bucket type energy dissipators below spillways and outlet works founded on rock.

2 REFERENCES

The standards listed below contain provisions which, through reference in this text, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards listed below:

IS No.	*Title*
1786 : 2008	Specification for high strength deformed steel bars and wires for concrete reinforcement (*fourth revision*)
4410 (Part 18) : 1983	Glossary of terms relating to river valley projects: Part 18 Energy dissipator devices (Stilling basins)
4997 : 1968	Criteria for design of hydraulic jump type stilling basins with horizontal and sloping apron
5186 : 1994	Criteria for design of chute and side channel spillways (*first revision*)
7365 : 2010	Criteria for hydraulic design of bucket type energy dissipators (*second revision*)
11772 : 2009	Guidelines for design of drainage arrangements of energy dissipators and training walls of spillways (*first revision*)

3 TERMINOLOGY/DEFINITIONS

For the purpose of this standard, the definitions given in IS 4410 (Part 18), IS 4997 and IS 7365 shall apply. Definition sketch is given in Fig. 1.

4 STRUCTURAL DESIGN OF STILLING BASIN FLOOR

4.1 General

The basin floor elevation is generally decided on the basis of foundation conditions and the length of the basin is decided on the basis of hydraulic considerations in accordance with IS 4997. The width depends on the number of openings and piers on spillway crest.

4.2 Structural Design

The basin floor slab is subjected to uplift, pounding and vibrations from hydrodynamic forces in the hydraulic jump. On yielding foundation it may suffer differential settlement. Therefore, the basin floor slab shall be designed for the stresses induced due to above forces.

The important elements of the structural design of basin floor is determination of thickness of the floor and details of anchorages in the bed rock. Uplift force is jointly resisted by the self weight of floor and contribution from the anchorages. For calculations of uplift, a thickness of the floor is assumed initially, and is confirmed by detailed calculations.

4.2.1 *Minimum Thickness*

The minimum thickness of the floor slab depends on the foundation conditions and the magnitude of uplift forces. A slab of about 600 mm thickness is the minimum recommended. Actual slab thickness shall be determined by analysing uplift and differential movement.

4.3 Design Procedure

The structural design of floor slab consists of the following steps:

a) Determination of uplift force;
b) Details of anchorages into bed rock, that is, diameter, number and spacing of anchor bars;
c) Determination of length of anchors considering dislodgement of rock mass against uplift force; and
d) Working out details of slab reinforcement.

4.3.1 *Uplift Force*

Uplift of the concrete lining of the stilling basin could be caused due to one or a combination of the following:

a) *Hydrostatic uplift*—The uplift caused by seepage gradient below the stilling basin.

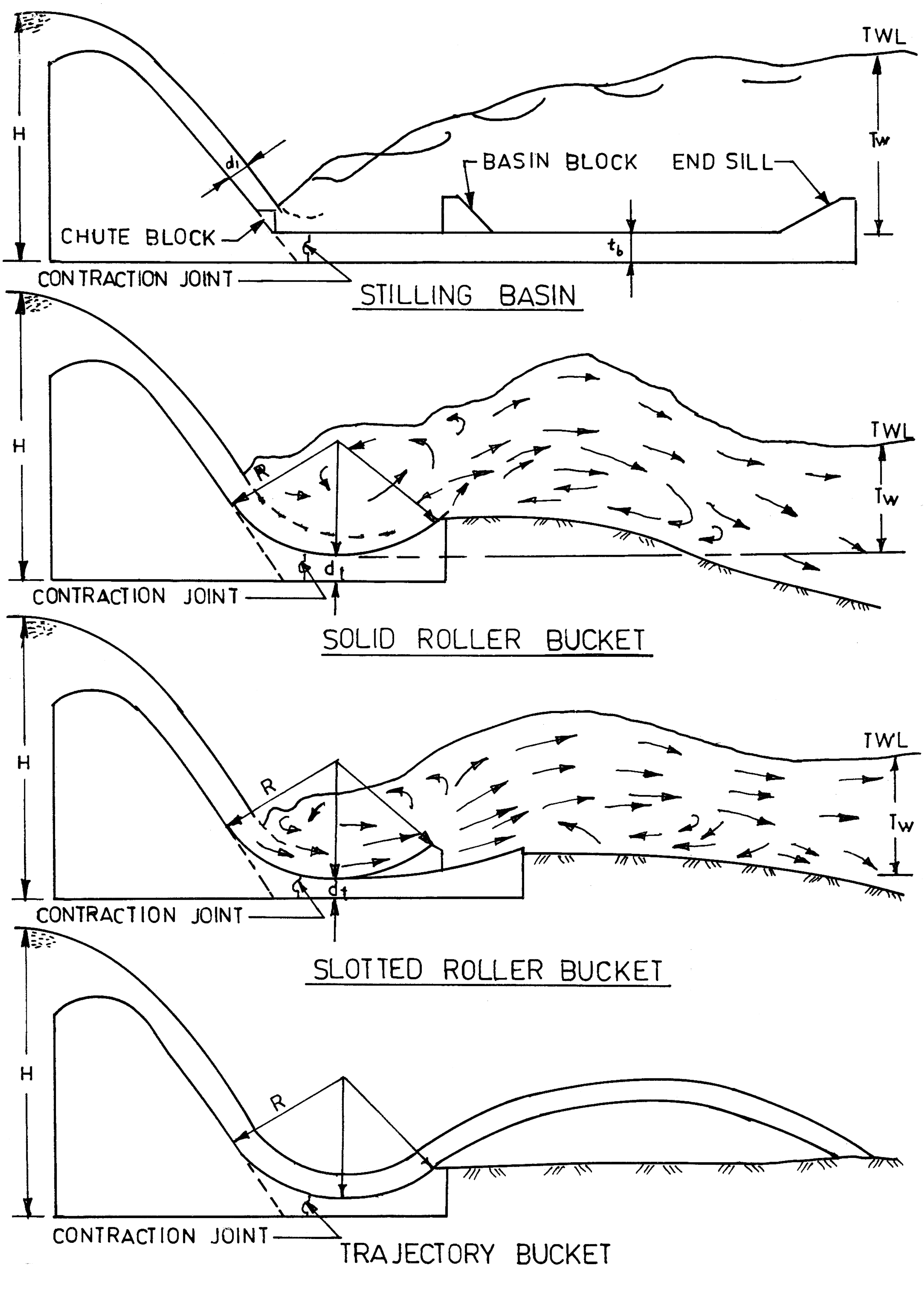


FIG. 1 DEFINITION SKETCH

b) *Hydrodynamic uplift*—The propagation of fluctuating pressures in the hydraulic jump through unsealed joints or cracks in the floor such that mean pressure prevails at the interface of the concrete and bed rock and hence hydrodynamic uplift is caused whenever instantaneous pressure on the upper surface is less than mean value of the fluctuating pressure.

4.3.1.1 *Hydrostatic uplift*

Provision of effective drainage system below the basin is necessary (*see* IS 11772). In view of the drainage arrangement provided below the basin, it may be adequate to assume 50 percent relief in the uplift pressures, as indicated below. Following three conditions may prevail and the most critical of them should be determined.

a) *Condition* I

Stilling basin operating during spillway design flood is shown in Fig. 2. The water surface over the slab at hydraulic jump profile for design discharge conditions.

Unbalanced uplift pressure,

$$u = 0.5\ (T_w \times W + t_b \times W) - (D_1 \times W + t_b \times W_c)$$

where

T_w = maximum tail water depth,
W = unit weight of water,
t_b = thickness of stilling basin floor,
W_c = unit weight of concrete, and
D_1 = depth of water in stilling basin.

b) *Condition* II

Reservoir at FRL with gates closed and stilling basin empty, that is, dewatered (*see* Fig. 3).

Unbalanced uplift pressure,

$$u = 0.5\ (T_{w1} \times W + t_b \times W) - (t_b \times W_c)$$

where

T_{w1} = minimum tail water depth.

c) *Condition* III

Sudden drawdown condition after the design flood has just passed over the spillway and the gates are closed again.

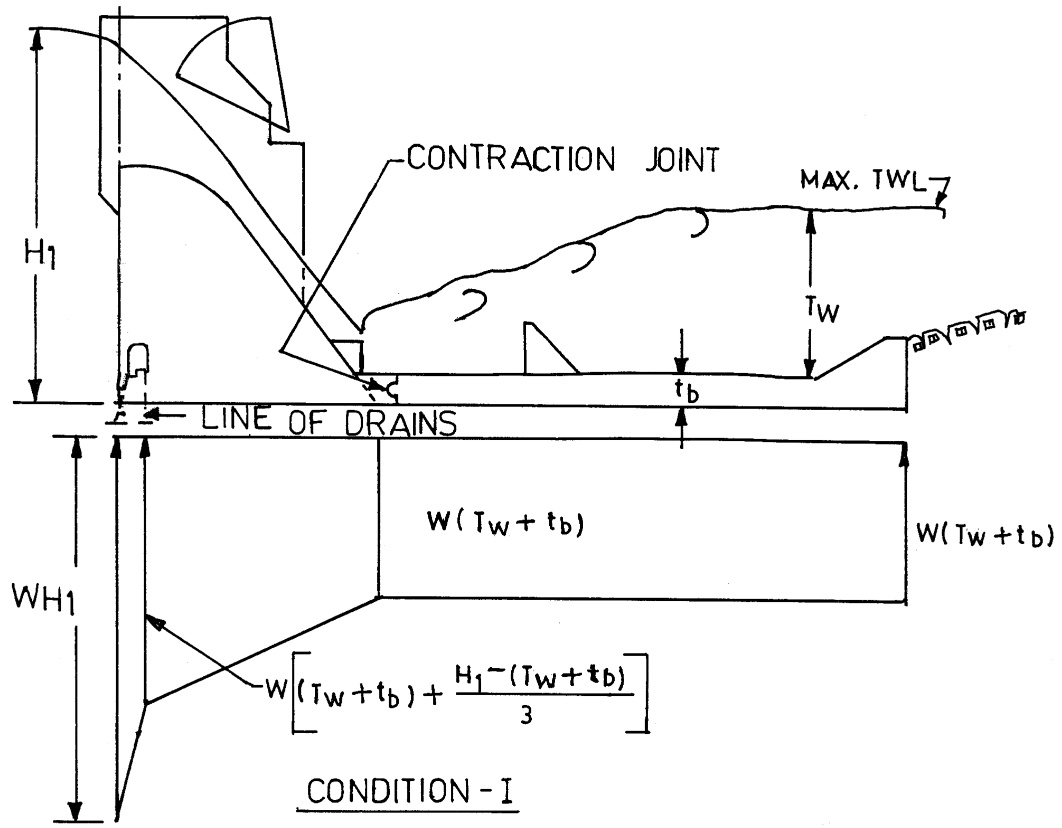


FIG. 2 SPILLWAY OPERATING AT DESIGN FLOOD (UPLIFT DIAGRAM)

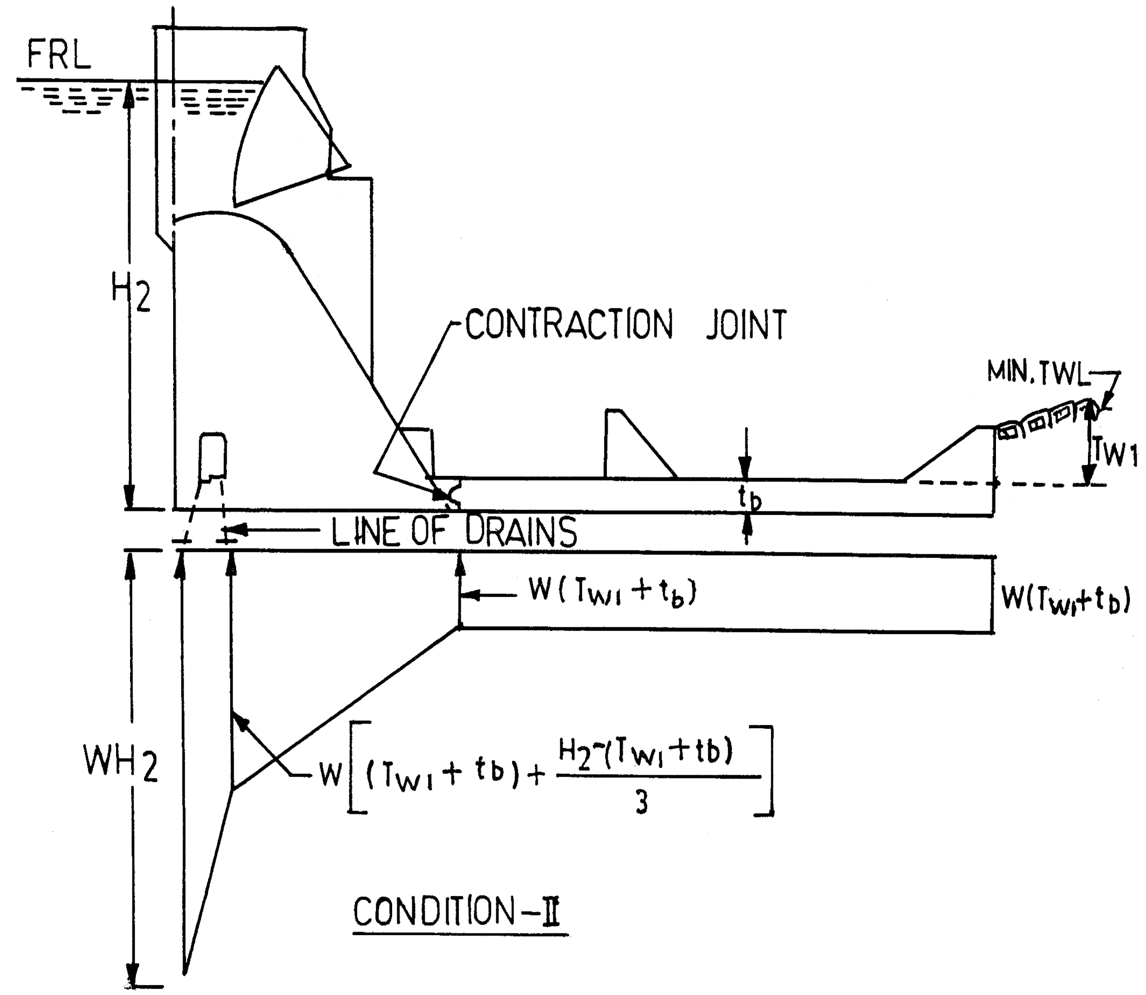


FIG. 3 RESERVOIR AT FRL WITH GATES CLOSED AND STILLING BASIN EMPTY (DEWATERED) (UPLIFT DIAGRAM)

In this case the uplift below the basin may be taken the same as the condition I and the water load in the basin be taken as the minimum TWL which may be the deepest river bed level, minimum water level in the river from power/other considerations, etc (*see* Fig. 4).

Unbalanced uplift pressure,

$$u = 0.5\ (T_w \times W + t_b \times W) - (T_{w1} \times W + t_b \times W_c)$$

4.3.1.2 *Hydrodynamic uplift*

The hydrodynamic uplift pressure F'_m over the slab monolith according to clause **4.3.1.** is given by:

$$F'_m = K\phi_p.\phi_1.\phi_2\left(\frac{\rho.V_1^2}{2}\right)$$

where

K = factor accounting for the probability of occurrence of fluctuating pressures.
= 3.09 corresponding to 99.8 percent probability of occurrence;

ϕ_p = pressure coefficient given in Fig. 5;

ρ = mass density of water, 1 000 kg/cum;

V_1 = velocity of flow entering the stilling basin;

L_1 = length of the monolith in flow direction;

L_2 = width of the monolith perpendicular to flow direction;

ϕ_1 = coefficient of correlation of pressure along L_1 (*see* Fig. 5); and

ϕ_2 = coefficient of correlation of pressure along L_2 (*see* Fig. 5).

The values of pressure coefficient for various froude numbers shall be read out from Fig. 5.

The largest of the above values of uplift pressure shall be considered for design purpose.

4.3.2 *Design of Anchors*

Generally the diameter of the anchor bar should at least be 25 mm. The diameter of the hole into which the anchors are placed and grouted should not be less than

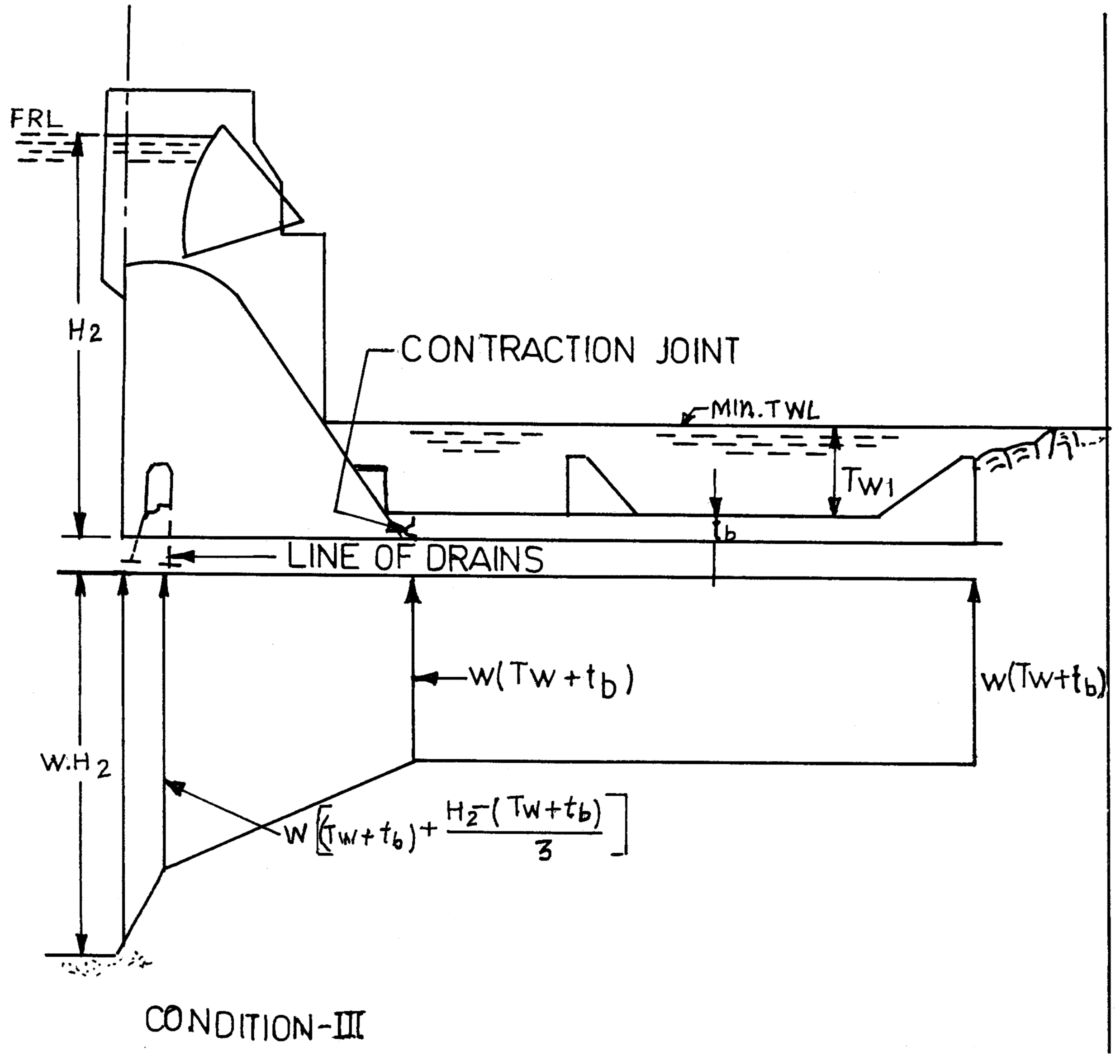


FIG. 4 SUDDEN DRAWN CONDITION (UPLIFT DIAGRAM)

1.5 times the diameter of the anchor bar. The other details shall be worked out as shown below:

Number of anchors (n) required per unit area is given by :

$$n = \frac{\text{Unbalanced uplift pressure}}{a \times \sigma_{st}}$$

where

a = cross-sectional area of the bar; and

σ_{st} = permissible tensile stress of steel, in kN/m^2.

Number of bars to be rounded to the next higher integer

$$\text{Spacing of bars} = \sqrt{\frac{1}{n}}$$

$$\text{Actual force in each anchor} = \frac{\text{Unbalanced uplift pressure}}{\text{No. of anchors required per unit area}} = \frac{u}{n}$$

$$\text{The depth of anchor bars in rock mass} = \frac{\text{Actual force in each anchor}}{\pi \times d_b \times F_{b1}}$$

Or

$$\frac{\text{Actual force in each anchor}}{\pi \times d_a \times F_{b2}}$$

where

d_b = diameter of anchor bar,

d_a = diameter of anchor hole,

F_{b1} = permissible bond stress between steel and grout, and

F_{b2} = permissible bond stress between grout and rock.

The greater of the two should be adopted as anchor depth. The value of the permissible bond stress would

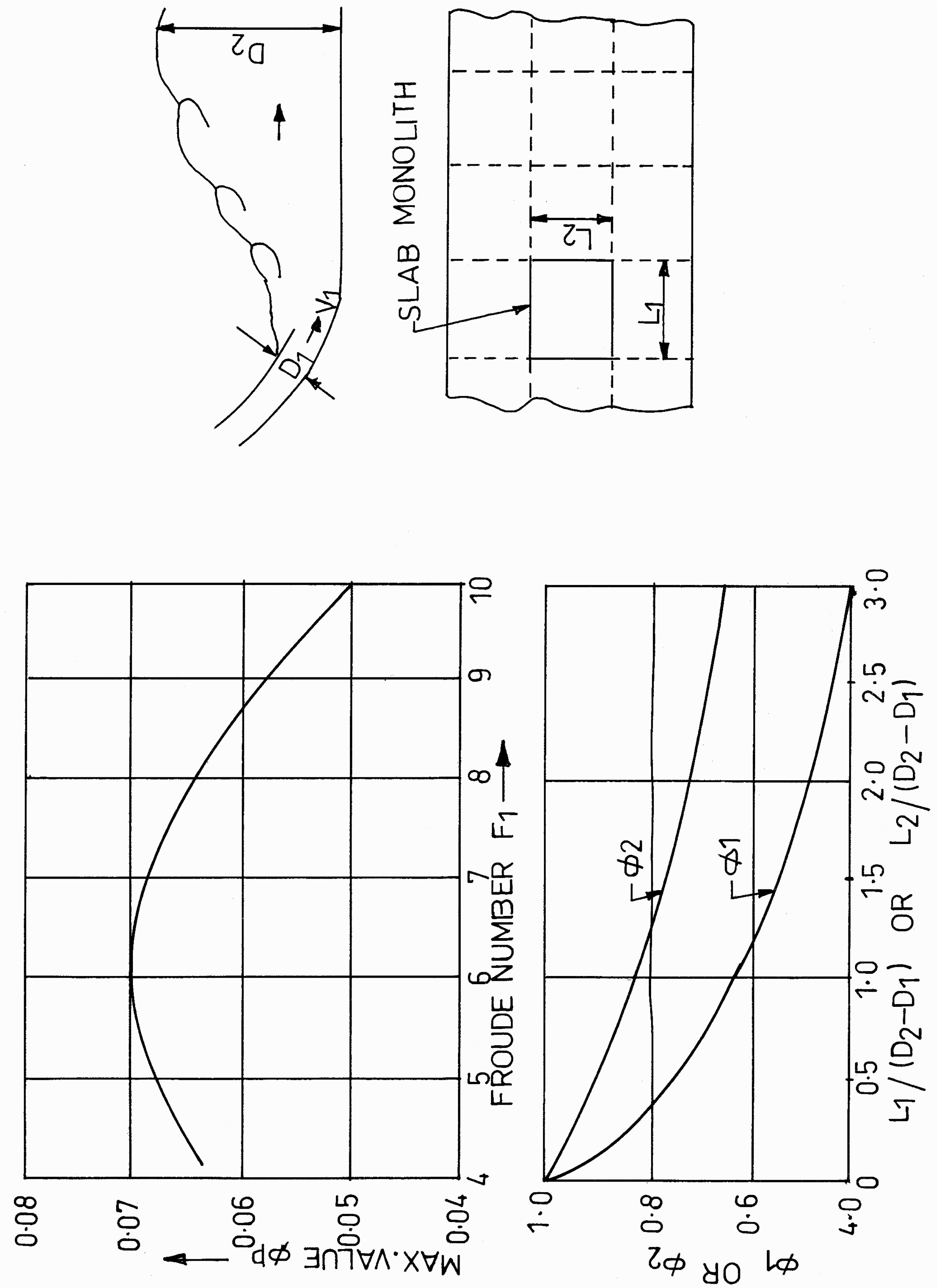


FIG. 5 DESIGN CHARTS FOR HYDRODYNAMIC UPLIFT

vary for different site conditions and proportion of grouts. In the absence of data, the following values for 1 : 2 ratio proportion of grout may be adopted.

F_{b1} = 600 kN/m^2

F_{b2} = 400 kN/m^2

Bond length should be checked for bond between steel and grout and also for bond between rock and grout.

4.3.3 *Check for Dislodging of Rock Mass Anchored Against Uplift Pressure*

The depth of anchor bar in the rock mass calculated in **4.3.2** should be checked for dislodgement of rock mass against uplift pressure. For this, the depth of anchor bar in the rock mass should be sufficient to engage a conical mass of rock with a vertex angle of 45°, the weight of which should be able to withstand the net upward force (*see* Fig. 6).

The depth of anchor required from the above considerations shall be checked as follows:

a) Depth of conical mass of rock at bottom, h_1

$$= \frac{(\text{Spacing of anchor bar})/2}{\tan 22.5^\circ} \text{m}$$

b) Weight of conical mass at bottom

$$= \frac{\pi}{4} \times (2 \times h_1 \times \tan 22.5^\circ)^2 \times \frac{h_1}{3} \times W_{rs}$$

where

h_1 = depth of anchor bar in conical portion (*see* Fig. 6); and

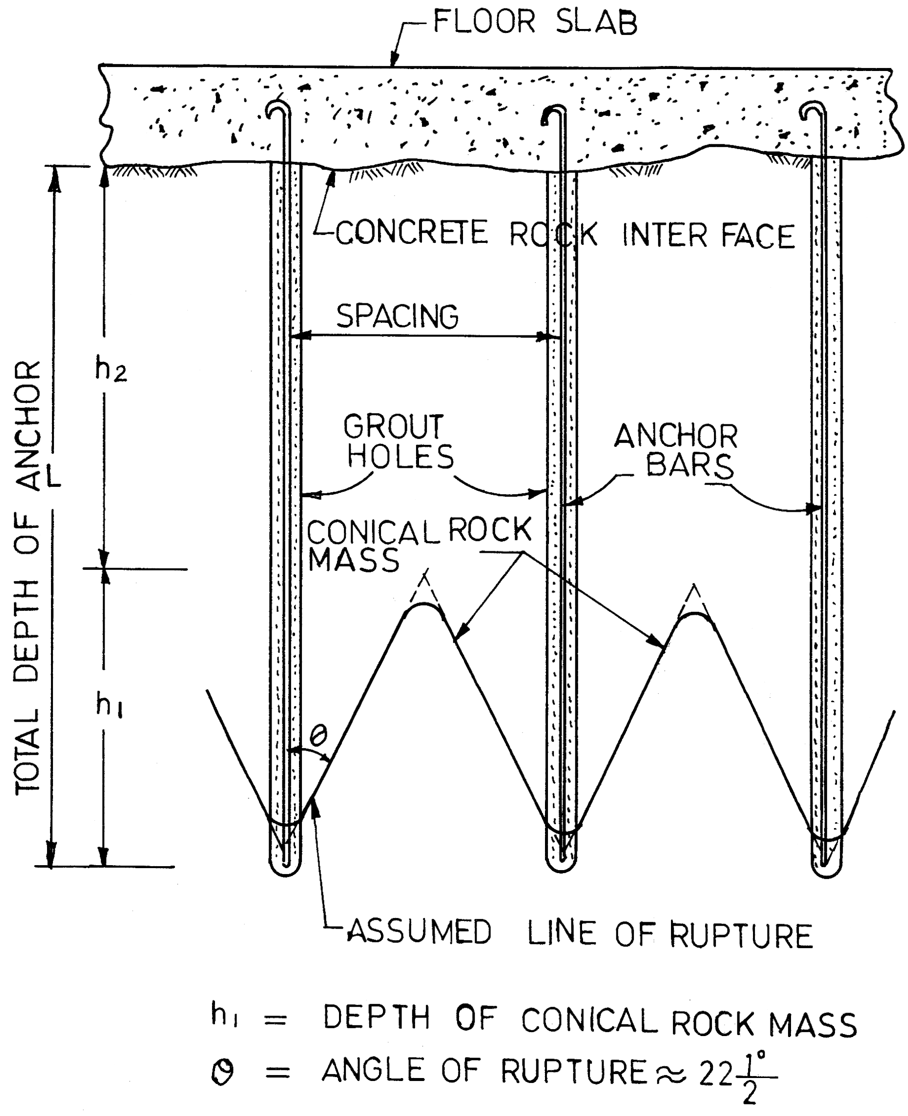


FIG. 6 DETAILS OF ANCHORS AND GROUT HOLES

W_{rs} = Submerged unit weight of rock, in kN/m^3

c) Depth h_2 required excluding conical mass

$$h_2 = \frac{\left[\begin{array}{c}\text{Actual force on each anchor} - \text{Self} \\ \text{weight of slab covered by} \\ \text{each anchor} - \text{Weight of conical mass}\end{array}\right]}{(\text{Spacing})^2 \times \text{Submerged weight of rock}}$$

NOTE — If the governing uplift considered is hydrostatic (that is, conditions (a) to (c), then the term representing self weight of slab should be omitted from the above equation.

d) Total depth of anchor $L = h_1 + h_2$

The above procedure assumes that no tension is permissible in the foundation rock. However, in sound and hard rock some tension is allowed to reduce the depth of anchor.

It should also be ensured that the depth of anchor bar as calculated in **4.3.2** is not less than the value of '*L*' calculated using the above equations. Not withstanding the results of above calculations, a minimum 3 m length of anchor should be provided.

The calculations should be refined to obtain a satisfactory combination of the values of slab thickness and anchor bars, consistent with the characteristics of the bed rock.

Annex A gives an example to illustrate the methods of calculation.

4.4 Basin Floor Slab

4.4.1 *Floor Slab Thickness*

The thickness of floor slab depends on the foundation conditions and magnitude of uplift forces. A slab of about 600 mm thickness is the minimum recommended. Actual slab thickness needed shall be determined by analysing hydrostatic uplift and differential foundation movement.

4.4.2 *Floor Slab Reinforcement*

4.4.2.1 In thick slabs on rock foundations normally covered with nominal tail water, structural reinforcement is not necessary except in the appurtenances of the stilling basin. Uplift on a slab should be taken care of by adequate anchors. The slab is divided into independent panels by contraction joints parallel and perpendicular to channel or basin centre line to avoid serious shrinkage and temperature cracking with the use of nominal reinforcement which does not extend across the joints. Size of panel should be large enough to resist distorting hydrodynamic forces. Panels should be cast in alternate bays with construction joints.

4.4.2.2 The independent panels of slab are reinforced with nominal steel (*see* IS 1786) to prevent harmful cracking resulting from shrinkage and temperature stresses not relieved by contraction joint and on yielding foundations to avoid possible cracking from differential settlement. Usually, a slab on unyielding foundation is reinforced in the top face only because bond between the concrete and rock at the bottom is relied on to distribute shrinkage cracks and to minimize bending stresses in the anchored slab for the assumed uplift head. The minimum amount of reinforcement for independent panels on unyielding rocks is 20 mm diameter bars at 300 mm centre-to-centre both ways. Additional reinforcement should be provided for unfavourable foundation condition or for high hydrostatic uplift pressure.

4.4.3 *Differential Movement*

On relatively yielding rock foundations, the independent floor panels are subject to possible differential movement of adjacent blocks and a key at each transverse contraction joint (extending into foundation under the joint attached to the slab downstream and supporting the slab upstream from the joint) may be required to prevent the downstream side of a joint from being raised above the upstream side as water at high velocity striking such a projection would increase the hydrostatic pressure in the joint and hence the uplift under the slab. Higher the velocity, more serious will be the condition resulting from such relative movement. The keys also increase resistance to possible movement and serve as seepage cutoffs downstream from transverse drains. Details of key are covered in IS 5186.

4.4.4 *Reinforcement Cover*

Chute floor and stilling basin slab shall have minimum 100 mm cover for reinforcement.

4.5 Basin Blocks (Structural Provision)

4.5.1 *General*

Location and optimum shape of basin blocks shall be decided on the basis of IS 4997. The dimensions of the basin blocks are shown in Fig. 7. The purpose of the block is to dissipate energy and thereby to reduce the length of basin.

h_b = height of basin block,

S_b = spacing between the blocks,

W_b = width of block, and

D_2 = conjugate depth.

4.5.2 *Forces on Basin Blocks*

Dynamic force against the upstream face of the basin blocks is approximately that of a Jet impinging upon a

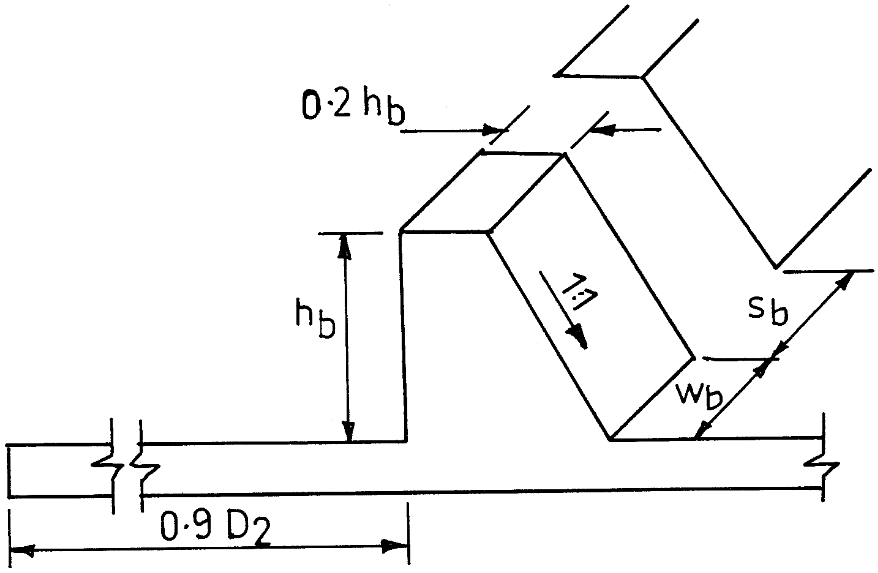


FIG. 7 BASIN BLOCK

plane normal to the direction of flow (*see* Fig. 8).

Force P acting at $h_b / 2 = 2WA\ (D_1 + h_{v1})$

where

W	=	unit weight of water,
A	=	area of upstream face of block,
h_b	=	height of basin block, and
$(D_1 + h_{v1})$	=	specific energy of the flow entering the basin.

4.5.2.1 Negative pressure on the back face of the blocks will further increase the total load. However, this may be neglected if above equation is used. Basin block is to be designed as cantilever as shown in Fig. 8.

4.5.3 *Reinforcement*

The reinforcement shall be calculated by the following formula and placement of the reinforcement is shown in Fig. 9.

$$\text{Area of steel } A_{st} = \frac{M}{\sigma_{st}\, jd}$$

where

M = moment due to force P (*see* **4.5.2**),

$$= P \times \frac{h_b}{2}$$

h_b = height of basin block,

σ_{st} = permissible tensile stress of steel, and

d = effective depth of block.

NOTES

1 The basin block is tied into the floor slab by reinforcing steel.

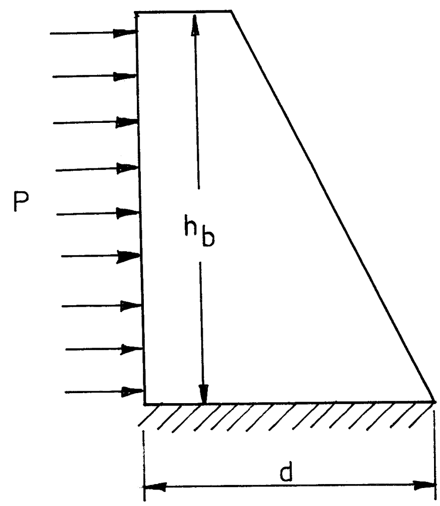


FIG. 8 FORCES ON BASIN BLOCK

2 All reinforcing steel in a basin block is placed minimum 150 mm from the exposed surface because of the possible erosive and cavitation action of the high velocity currents.

4.6 Chute Blocks

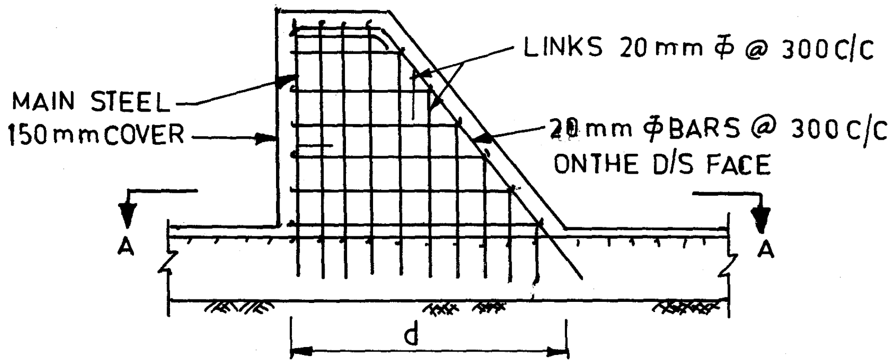
Nominal reinforcement of 20 mm diameter bar at 300 mm centre-to-centre both ways may be provided on all exposed faces duly anchored in apron concrete.

5 SPILLWAY BUCKET REINFORCEMENT

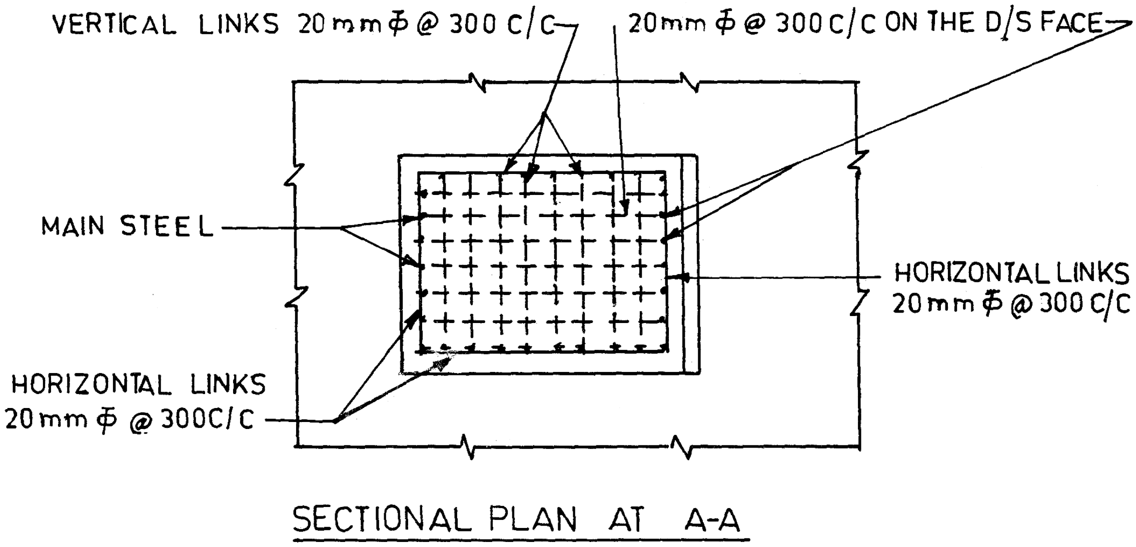
5.1 Trajectory/Solid Roller Bucket (*see* Fig. 10 and Fig.11)

5.1.1 *Forces and Moments*

Horizontal force on the bucket is due to change in



9A Vertical Section Showing General Arrangement of Reinforcement



9B Sectional Plan at A-A

FIG. 9 REINFORCEMENT ARRANGEMENT

momentum and is given by the following formulae:

$$\text{Total horizontal force per metre width on the lip } (F) = \frac{WqV}{g}\left(1-\cos\theta_e\right)$$

where

W = Unit weight of water,

q = Discharge intensity,

V = Velocity of flow, and

θ_e = Exit angle of the bucket.

Moment of the horizontal force about horizontal plane A-A passing through invert of the bucket (*see* Fig. 10) is given by:

$$M = \frac{F \times R\ (1-\cos\ \theta_e)}{2}$$

where

R = radius of bucket.

Effective depth '*d*' of bucket for resisting moment M may be taken as;

$$d = \sqrt{R^2 + (\text{R sin } \theta_e + t_w)^2} - R - \text{effective cover,}$$

where t_w is the width of bucket lip.

5.2 Reinforcement

Area of the steel A_{st} to resist moment M is given by.

$$A_{st} = \frac{M}{\sigma_{st}\ jd}$$

Provided minimum steel (along flow) = 20 mm diameter bar at 300 mm centre-to-centre.

Provide distribution steel = 20 percent of main steel with a minimum of 16 mm diameter bar at 300 mm centre-to-centre.

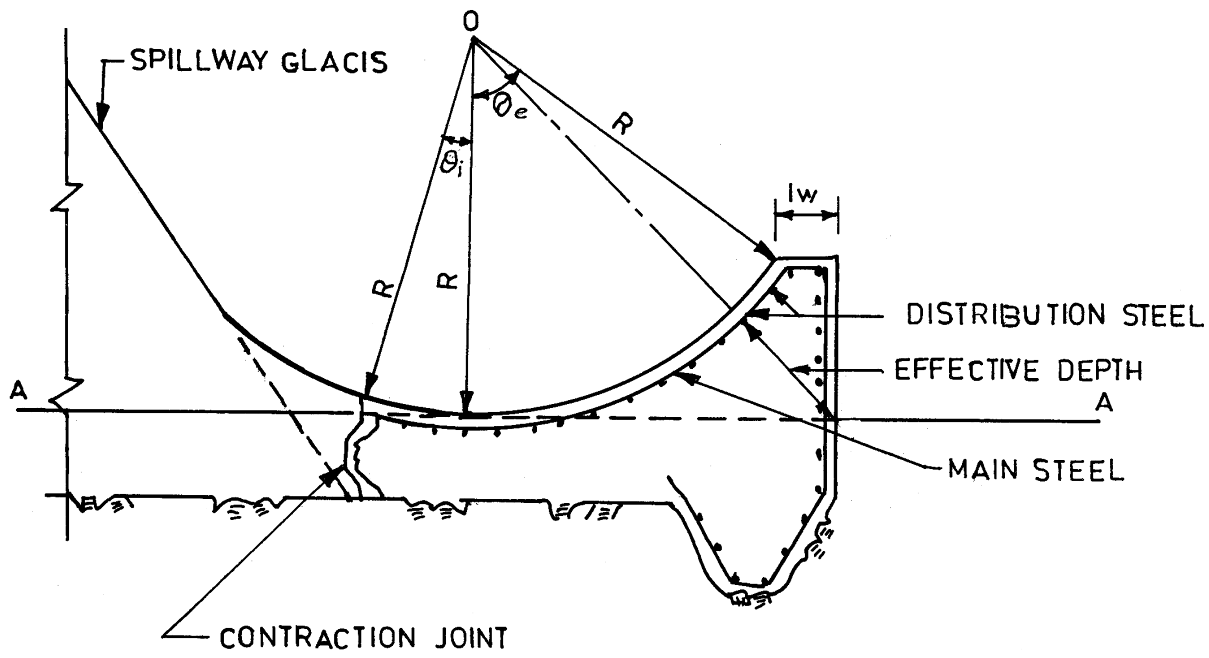


FIG. 10 TYPICAL SECTION OF A BUCKET

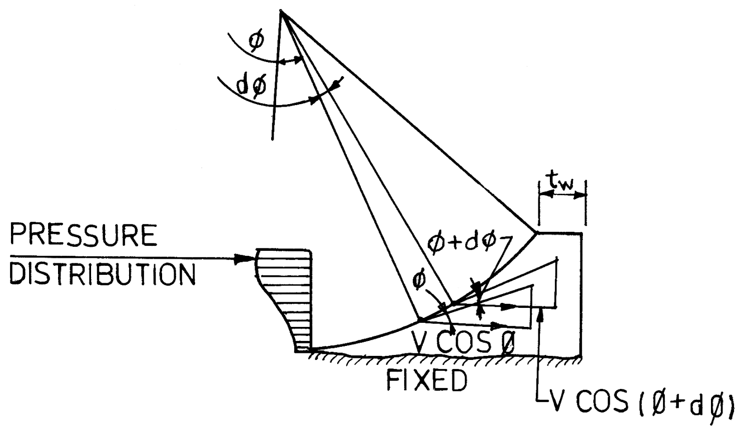


FIG. 11 FORCES ON THE BUCKET DUE TO MOVING WATER

5.3 Anchorage of Spillway Bucket (*see* Fig. 12)

5.3.1 The following three conditions may prevail:

a) *Condition* I — Spillway operating condition at design flood.
b) *Condition* II — Reservoir at FRL with gates closed and the bucket empty (dewatered).
c) *Condition* III — Sudden drawdown condition.

Generally the design is carried out considering uplift below the bucket corresponding to maximum TWL (condition 1 and condition 3) ignoring the weight of water in the bucket and lip portion of the bucket on conservative side. The methodology generally adopted is given in **5.3.3**. However, the weight of water in the bucket may be suitably considered in design when it is likely to have a substantial effect.

5.3.2 Provision of effective drainage system below the bucket is essential (*see* IS 11772). In view of the drainage arrangements below the bucket, it may be adequate to assume 50 percent relief in uplift pressures.

5.3.3 Upward force per unit area F_u (*see* Fig. 12) is given by the formula:

$$F_u = R\left[0.5\beta W - W_c\alpha - W_c\right.$$

$$\left\{1-\frac{0.25(\sin 2\theta_e+\sin 2\theta_i)+0.5(\theta_e+\theta_i)}{(\sin\theta_e+\sin\theta_i)}\right\}\Bigg]$$

θ_i = inlet angle of bucket

Number of anchors per unit area = $n = \dfrac{F_u}{a\sigma_{st}}$

and spacing of anchors = $\sqrt{\dfrac{1}{n}}$

Anchors are generally staggered in plan.

5.3.4 *Depth of Anchors in Rock Mass*

The depth of anchors should be determined by considering failure between anchor bar and grout and also between grout and rock as given in **4.3.2**.

6 SLOTTED ROLLER BUCKET (STRUCTURAL PROVISION)

6.1 *General*

6.1.1 Dimensions of the slotted roller bucket should be worked out on the basis of IS 7365. For anchor design **5.3** will be applicable. Provision of effective drainage system below the bucket is essential as given in IS 11772.

Some salient dimensions to be used in this standard are (*see* Fig.13 and Fig.14):

b = 0.125 R

b_2 = 0.05 R

L_a = R sin (θ_e – 8) + 0.05 R = R [sin (θ_e – 8) + 0.05]

b_1 = b_2 + 2L_a tan 2°

b' = b – 2L_a tan 2°

6.1.2 Discharge/metre is calculated by $\theta = Q/L$ m³/s/m

where

Q = total design discharge at MWL, in m³/s; and

L = length of spillway in m.

Velocity is given by the equation;

$V = \sqrt{2gH}$ m/s

where

H = fall of water (head) from MWL to bucket invert.

6.2 Design of Reinforcement for Bucket Teeth

6.2.1 Force on bucket teeth above plane AB and in a direction parallel to it (*see* Fig. 13 and 14) is given by:

$$F_1 = \frac{WqV}{g}\left[1-\cos\left(\theta_e-8°\right)\right]\times b \text{ (Approximately)}$$

The teeth should be designed as a cantilever fixed at the plane AB.

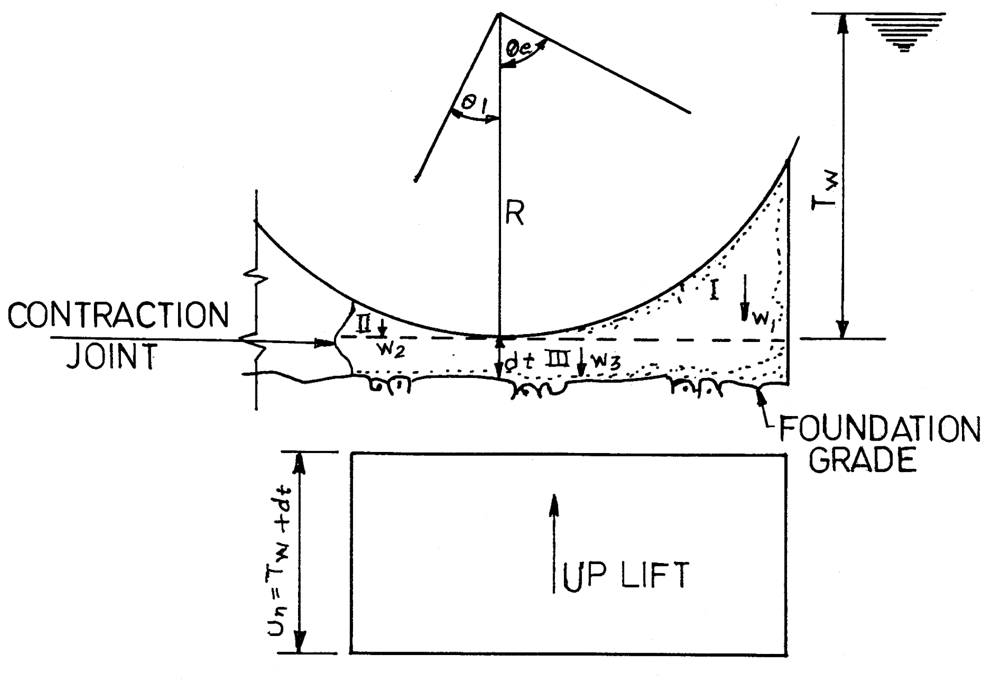


FIG. 12 UPLIFT AND BODY FORCES ON THE BUCKET

$$\text{Bending moment, } M = \frac{F_1 \times h_1}{2}$$

Where h_1 is defined in Fig. 14 and is given as $h_1 = R[1 - \cos(\theta_e - 8°)]$

6.2.2 Area of main steel A_{st} is given by:

$$A_{st} = \frac{M}{\sigma_{st}\, jd}$$

where

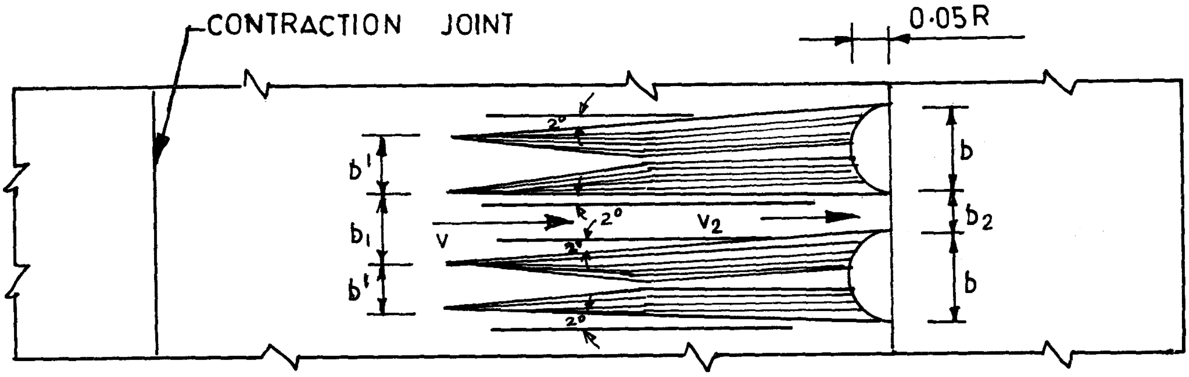


FIG. 13 PLAN SHOWING TEETH OF SLOTTED ROLLER BUCKET

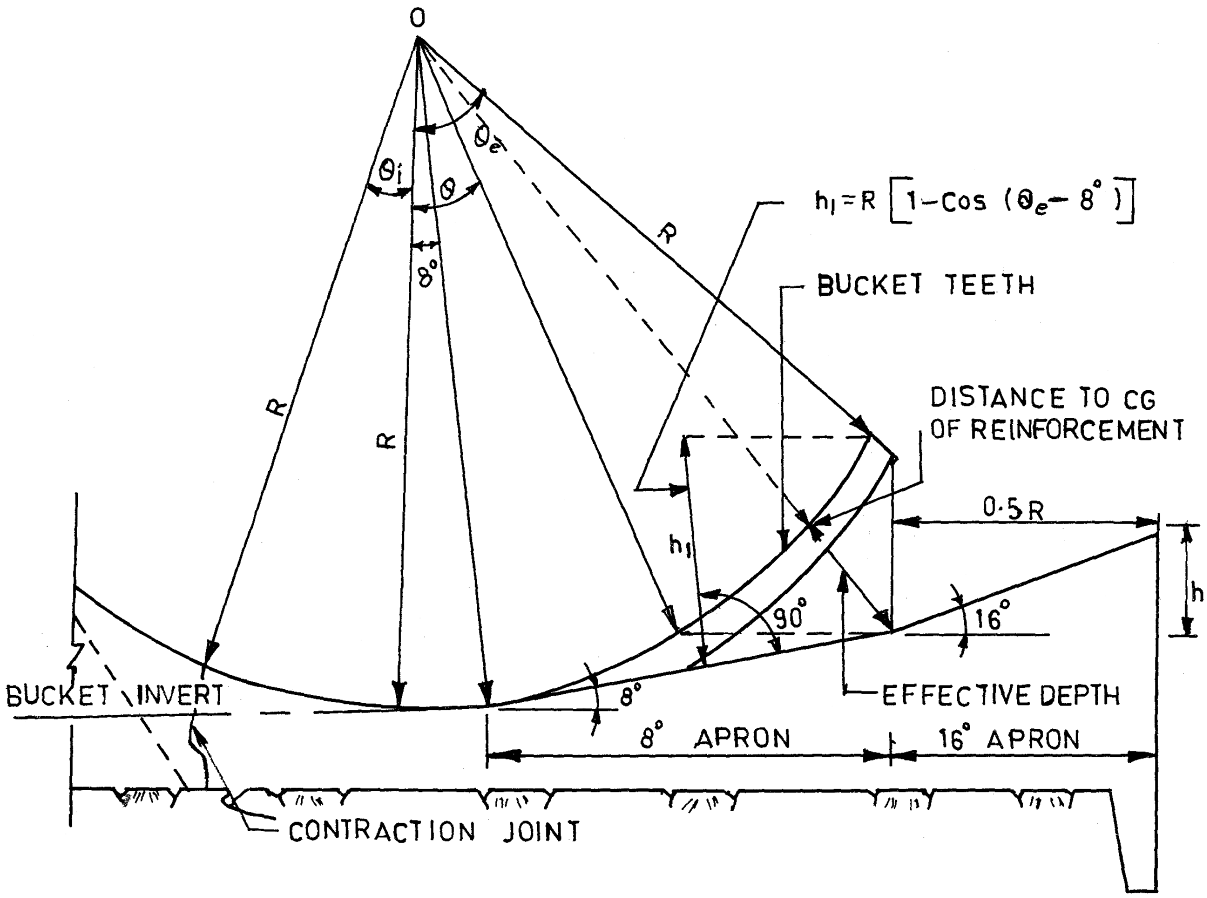


FIG. 14 DEFINITION SKETCH OF BUCKET TEETH OF SLOTTED ROLLER BUCKET

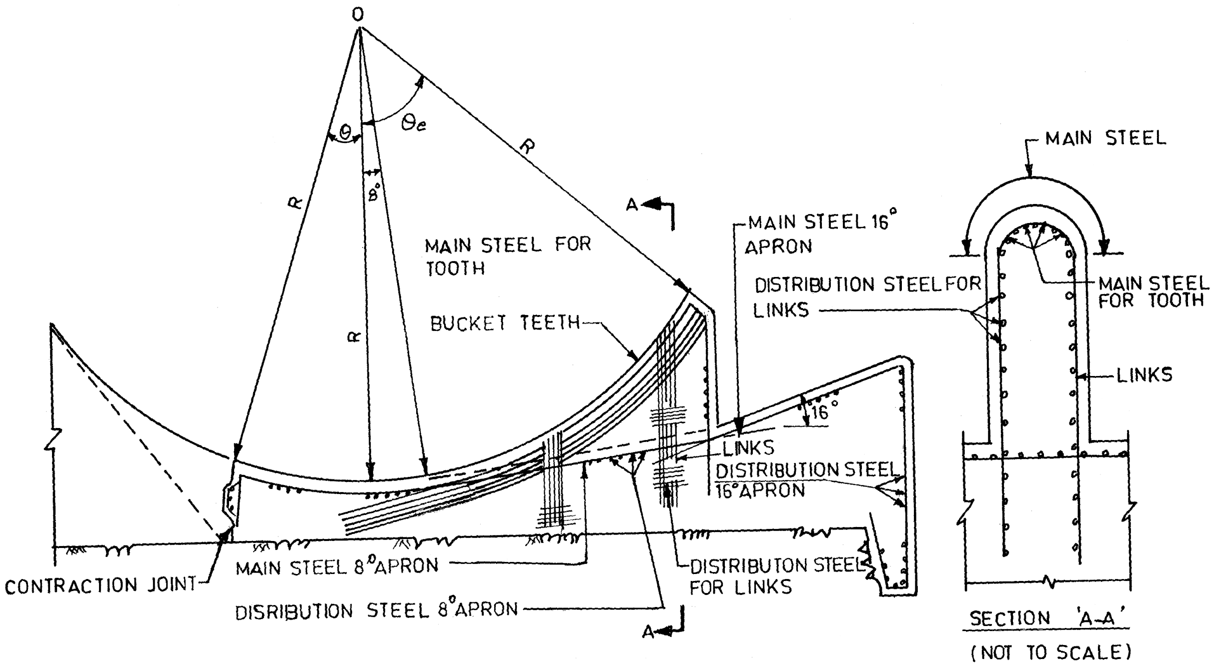


FIG. 15 REINFORCEMENT DETAILS FOR SLOTTED ROLLER BUCKET AND APRONS (AT CENTRE LINE OF TEETH)

$d = \sqrt{(R\cos\theta)^2 + (R\ \sin\theta_e + 0.05\ R)^2} - R$ – effective cover.

$\cos\theta$ = [Radius — (Elevation of junction of 8° and 16° aprons — invert level)]/Radius (*see* Fig. 15).

j = lever arm factor and θ, θ_e and R are indicated in Fig. 14.

The main reinforcement should be provided along the curve as shown in Fig. 15. The minimum clear cover in the radial direction should be 80 mm. It shall be ensured that minimum reinforcement per tooth is 7 numbers 20 mm diameter bar.

6.2.3 *Design of Links for Tooth*

Provide 20 mm diameter link reinforcement at 300 mm centre-to-centre around tooth in the direction perpendicular to flow. Distribution steel for links shall be provided on three side faces of tooth and shall be 20 mm diameter bar at 300 mm centre-to-centre.

6.3 Design of Reinforcement for 8° Apron

Provide nominal reinforcement are as under:

a) Along flow (main steel) = 20 mm diameter at 300 mm centre-to-centre; and
b) Perpendicular to flow (distribution steel) = 16 mm diameter at 300 mm centre-to-centre.

6.4 Design of Reinforcement for 16° Apron

Horizontal force on 16° apron.

$$F_2 = \frac{WqV_2}{g}\left(\cos 8° - \cos 16°\right)$$

where

$$V_2 = \frac{b_1 V}{b_2}$$

b_1 = width of slot at entry, and

b_2 = width of slot at exit.

The horizontal force on apron is due to change in direction as above and it acts $\frac{h_2}{2}$ above apron level where h_2 = height of 16° apron.

$$\text{Bending moment} = F_2 \times \frac{h_2}{2}$$

$$\text{Area of steel } A_{st} = \frac{B.M}{\sigma_{st}\ jd_2}$$

where

d_2 = effective depth for 16° apron = $h_2 \cos 16°$– cover

NOTE — Minimum steel shall be provided as mentioned in **6.3**.

6.5 Sample calculations for slotted roller bucket are given in Annex B for guidance.

ANNEX A
(*Clause* 4.3.3)

ILLUSTRATIVE EXAMPLE FOR DESIGN OF SLAB AND ANCHORAGE

A-1 DATA GIVEN

a) D_1 = 2.43 m
b) V_1 = 46.1 m/s
c) F_1 = 9.44
d) T_W = 31.25 m

(a) to d): For the design discharge condition)

e) T_{W1} = 6 m corresponding to min power house flow
f) Size of the slab monolith L_1 = 16 m, L_2 = 14 m
g) ρ = 1 000 kg/m^3 (Mass density of water)
h) W = 9.81 kN/m^3 (Specific weight of water)
j) W_c = 22.56 kN/m^3 (Specific weight of concrete)
k) W_{rs} = 13.73 kN/m^3 (Specific weight of rock under submerged condition).
m) F_{b1} = 600 kN/m^2
n) F_{b2} = 400 kN/m^2
p) σ_{st} = 23 × 10^{-4} kN/m^2

Step 1: Calculate the design uplift pressure

Hydrostatic uplift

Assume slab thickness t_b = 2 m

Condition I: Spillway design flood

$$u = 0.5\,(31.25 \times 9.81 + 2 \times 9.81) - (2.43 \times 9.81 + 2 \times 22.56)$$
$$= 94.13 \text{ kN/m}^2$$

Condition II: Stilling basin empty

$$u = 0.5\,(6 \times 9.81 + 2 \times 9.81) - (2 \times 22.56)$$
$$= 39.24 - 45.12 = (-)5.88 \text{ kN/m}^2$$. Hence there is no uplift

Condition III: Sudden drawdown following design flood

$$u = 0.5\,(31.25 \times 9.81 + 2 \times 9.81) - (6 \times 9.81 + 2 \times 22.56)$$
$$= 59 \text{ kN/m}^2$$

Hydrodynamic uplift

K = 3.09 for F_1 =9.44,

ϕ_p = 0.052 (*see* Fig. 5)

D_2 = T_w = 31.25 m

$D_2 - D_1$ = 31.25 – 2.43 = 28.82 m

for $\dfrac{L_1}{D_2 - D_1} = \dfrac{16}{28.82} = 0.555$,

ϕ_1 = 0.76 (*see* Fig. 5)

for $\dfrac{L_2}{D_2 - D_1} = \dfrac{14}{28.82} = 0.486$,

ϕ_2 = 0.91 (*see* Fig. 5)

Hydrodynamic uplift pressure

$$F'_m = 3.09 \times 0.052 \times 0.76 \times 0.91 \times \frac{1}{2} \times 1\,000 \times (46.1)^2$$
$$= 118.1 \text{ kN/m}^2$$

Therefore design uplift pressure = 118.1 kN/m^2

Step 2: Determine number of anchor bars/spacing

Try 25 mm dia bars.

Area = 49 × 10^{-4} m^2

No. of anchor bars per unit area $= \dfrac{118.1}{4.9 \times 10^{-4} \times 23 \times 10^4} = 1.047$

Say 1 bar/m^2

Spacing at 1 metre centre-to-centre

Therefore actual force in each anchor = 118.1 kN.

Step 3 : Determine depth of anchor bar

based on dia. of anchor bar

$$L = \frac{118.1}{\pi \times 0.025 \times 600} = 2.5 \text{ m}$$

based on dia of hole (40 mm) in the rock

$$L = \frac{118.1}{\pi \times 0.04 \times 400} = 2.35 \text{ m}$$

Step 4 : Check against dislodgement of rock mass

– Depth of conical rock mass at bottom (*see* Fig. 6)

$$h_1 = \frac{1/2}{0.414\,2} = 1.21 \text{ m}$$

– Weight of conical rock mass

$$= \frac{\pi}{4}(2 \times 1.21 \times 0.414\,2)^2 \times \frac{1.21}{3} \times 13.73$$
$$= 4.37 \text{ kN}$$

– Depth required excluding conical rock mass

$$h_2 = \frac{118.1 - (2 \times 22.56) - 4.37}{1^2 \times 13.73} = 4.99 \text{ , Say 5 m}$$

Therefore, total depth $L = h_1 + h_2$

$= 1.2 + 5.0$

Say = 6.2 m

The depth of anchorage will be 6.2 m

Results:

Size of the slab monolith = 16 m × 14 m

Thickness of concrete slab = 2 m

Dia. of anchor bar = 25 mm

Spacing = 1 metre centre-to-centre

Depth of anchorage = 6.2 m

NOTE — Any refinement required can be done by repeating the calculations with modified values of slab thickness and diameter of anchor bar, etc.

ANNEX B

(*Clause* 6.5)

SAMPLE CALCULATIONS FOR SLOTTED ROLLER BUCKET

B-1 DATA GIVEN

a) Exit angle = 45°
b) Length of bucket = 237.0 m
c) Discharge at MWL = 14 501 cumecs
d) Width of tooth of d/s end = 1.125 m
e) Radius of bucket = 9.0 m
f) Invert level of bucket = R.L. 403.50 m
g) Junction 8° and 16° apron = R.L. 404. 37 m
h) MWL = R.L. 434.800 m

$$\text{Discharge } q = \frac{14\ 501}{237} = 61.185 \text{ cumecs/m}$$

Say q = 62 cumecs /m

according to **6.1.2**

$$V = \sqrt{2\ gH}$$

$$= \sqrt{2 \times 9.81 \times 31.30} = 24.781 \text{ m/s}$$

where

H = Fall of water (head) from MWL to bucket invert (434.800 – 403.50) = 31.30 m

B-2 DESIGN OF REINFORCEMENT FOR BUCKET TOOTH

Force on tooth above plane AB in a direction parallel to it is given by:

$$F_1 = \frac{WqV}{g}\left[\left(1 - \cos\left(\theta - 8^\circ\right) \times b\right)\right]$$

$$= \frac{9.81 \times 62 \times 24.781}{9.81}[1 - \cos\ (45° - 8°)] \times 1.125$$

$= 348.05$ kN

Now,

h_1 (*See* Fig. 14) = R [1 – cos (θ_e – 8°)]

= 9 [1 – cos (45° – 8°)

= 1.812 3 m

$$\text{B.M.} = F_1 \times \frac{h_1}{2} \quad (\textit{see } \textbf{6.2.1})$$

$$= 348.05 \times \frac{1.812\ 3}{2}$$

= 315.385 kN.m

Here, cos θ (*see* Fig. 14)

$$= \frac{\text{Radius} - (\text{Elevation of junction of } 8^\circ \text{ and } 16^\circ \text{ aprons} - \text{invert level})}{\text{Radius}}$$

$$= \frac{9 - (404.37 - 403.5)}{9}$$

$= 0.903\ 3$

Effective depth

$$d = \sqrt{(R\ \cos\theta)^2 + (R\ \sin\theta_e + 0.05\ R)^2} - R - \text{effective cover}$$

= 152 cm = 1.52 m

Area of steel required

$$A_{st} = \frac{M}{\sigma_{st}\ jd} \quad (\textit{see } \textbf{6.2.2})$$

Assuming σ_{st} = 80% of 23 × 10^4 kN/m^2 for hydraulic structures

= 18.40 × 10^4 kN/m^2

$$A_{st} = \frac{315.385 \times 10^4}{18.4 \times 10^4 \times 0.8 \times 1.52} \text{cm}^2$$

$$= 14.095 \text{ cm}^2$$

Therefore, provide 20 mm diameter 7 numbers bars per tooth (minimum reinforcement).

B-2.1 Design of Links

Provide 20 mm diameter link reinforcement at 300 mm centre-to-centre around tooth in the direction perpendicular to flow. Distribution steel for links shall be provided on three side faces of tooth and shall be 20 mm diameter bar at 300 mm c/c.

B-3 DESIGN OF REINFORCEMENT FOR 8° APRON

Provide nominal reinforcement as under:

a) 20 mm diameter bar at 300 mm c/c main steel along flow; and
b) 16 mm diameter bar at 300 mm c/c perpendicular to flow.

B-4 DESIGN OF REINFORCEMENT FOR 16° APRON

Horizontal force on 16° apron

$$F_2 = \frac{WqV_2}{g}\left(\cos 8^{o} - \cos 16^{o}\right)$$

according to **6.4**

$$b_2 = 0.05\,R = 0.05 \times 9 = 0.45 \text{ m}$$

$$L_a = R \sin(\theta_e - 8) + 0.05\,R$$

$$= 9 \sin 37° + 0.45 \text{ m} = 5.86 \text{ m}$$

$$b_1 = b_2 + 2L_a \tan 2^{o} = 0.45 + 2 \times 5.86 \times \tan 2^{o}$$

$$= 0.86$$

$$V_2 = \frac{b_1 V}{b_2} = \frac{0.86 \times 24.78}{0.450} = 47.36 \text{ m/sec}$$

$$F_2 = 85.17 \text{ kN}$$

$$\text{B.M.} = F_2 \times \frac{h_2}{2}$$

where h_2 is rise of 16° apron = 0.5R tan16° =1.3 m (*see* Fig. 14)

$$\text{B.M.} = 85.17 \times \frac{1.3}{2}$$

$$= 55.36 \text{ kN.m}$$

$$A_{st} = \frac{B.M}{\sigma_{st}\ jd_2} \quad (\textit{see } \textbf{6.4})$$

where $d_2 = (h_2 \cos 16° - \text{cover}) = (1.25 - 0.1) = 1.15$ m

$$A_{st} = \frac{55.36 \times 10^4}{18.4 \times 10^4 \times 0.80 \times 1.15} = 3.27 \text{ cm}$$

Minimum steel provided :

a) Along flow = 20 mm diameter bar at 300 mm c/c; and
b) Perpendicular to flow = 16 mm diameter bar at 300 mm centre-to-centre.

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